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REPORT OF THE PORT AND
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Enhancement of Earthquake Resistance of Structures using Tire Chips as Compressible Inclusion

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Synopsis

An estimated 102 millions scrap tires are generated annually in Japan, 90% of which are recycled for thermal, material and retreading purposes. A major share of scrap tire recycling goes to the thermal recycling, a process that generates carbon dioxide, and is, thus contradictory to the principle of the Kyoto protocol that entered into the force since February 2005. However, the share of recycling as materials is still far from satisfactory. This research is an attempt to contribute towards material recycling of scrap tires by utilizing tire chips as a geomaterial for earthquake resistant reinforcement of geotechnical engineering structures. The objective of this research is to exploit the compressibility of tire chips by using such material as a cushion behind the massive port structures to reduce the load against the structures during the earthquake. Function of the sandwiched cushion layer is to provide flexibility, and thereby stability to a structure during the earthquakes by absorbing the energy. Thus, it is expected that only a part of the load coming to the structure will be transferred.

A series of shaking table tests of was performed on a model caisson by using a large underwater shaking table assembly. The seismic performance of the developed technique was verified by subjecting the soil-structure system into three different earthquake loadings and measuring the respective responses. The results demonstrate that the dynamic load against the caisson quay wall could be significantly reduced using the proposed earthquake resistant technique. Also, the presence of tire chips cushion could significantly reduce the earthquake-induced residual displacement of the caisson quay wall. In addition, owing to highly permeable nature of the tire chips cushion material, the pore water pressure building up in the immediate vicinity of the structure could be checked, and thus preventing the occurrence of liquefaction. Application of the developed technique, thus, not only contributes towards a better environment, but also provides a cost-effective design alternative for earthquake resistant design of port and harbor facilities.

Key Words: Tire chips, cushion buffer, earthquake resistant design, shaking table test, environment, cost-performance

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緩衝材としてタイヤチップを用いた構造物の耐震性評価

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要　旨

毎年確実に増加している日本の廃タイヤ（日本の人口にほぼ匹敵する本数）のリサイクル率は９０％である。2005年2月の京都議定書の発効によって，廃タイヤのリサイクルはサーマルリサイクルからマテリアルリサイクルへの転換（CO_2の排出量はサーマルリサイクルの約1/4に減少可能であるため）に迫られている．本研究は，マテリアルリサイクルの有効な利用方法の一つとして，タイヤチップ（廃タイヤを裁断したリサイクル品）を圧縮材として活用し，地震に対する社会基盤の安全かつ経済的な設計・施工と，廃棄物の再利用の両立を目指した耐震補強工法の開発を目的としている．

軽量で圧縮性および透水性に優れたタイヤチップを構造物の背面に緩衝材として使用することによって，構造物に柔軟性を与え、抗土圧構造物に作用する動的荷重の軽減工法を本研究で開発した．本研究では，圧縮性を有するタイヤチップクッションを用いたサンドイッチ型裏込め構造の地震時の性能に関して大型水中振動台を用いてケーソン式岸壁の振動実験を行った。実験では様々な地震波を用いて相互作用システムの耐震評価の検討を行った。

実験結果から，本研究で開発した耐震対策法を有するケーソン式岸壁に対して地震時の荷重を軽減できることが明らかになった。また，ケーソンの残留変位は耐震対策のない構造物に比べて小さくなる事が実証された．さらに，タイヤチップは粒状体材料であり地震時の液状化防止にも有効であることが明らかになった。本研究成果を用いることによって，社会基盤をより安全かつ経済的に設計・施工するだけではなく，廃タイヤのリサイクルはサーマルからマテリアルへの転換に対しても効果があることから，より良い環境づくりに貢献するという利点もある。従って、本工法は高い安全性と環境負荷縮減効果の両面を有するコストパフォーマンスが高いことから，港湾・空港構造物の耐震補強工法として有効な工法であると言える。

キーワード：　タイヤチップ，緩衝材，耐震設計，振動台実験，環境，コストパフォーマンス

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1. Introduction

The devastating earthquakes in the last one decade that struck Los Angeles, USA (January, 1994), Kobe, Japan (January, 1995), Kocaeli, Turkey (August, 1999), ChiChi, Taiwan (September, 1999), Bhuj, India (January, 2001), Bam, Iran (December, 2003), Niigata, Japan (October, 2004), Sumatra-Andaman (December, 2004) and most recently Kashmir region, India-Pakistan (October, 2005), bear evident to the fact that earthquakes are no longer a rare event in seismically active areas of the world. The deadliest Sumatra-Andaman earthquake on December 26, 2004 and the associated Tsunami in the Indian Ocean have wreaked havoc in many countries in South and Southeast Asia claiming thousands of human lives and bringing enormous damages to infrastructures of those countries. The devastating nature of these earthquakes only remind us once again that the destructive power of mother Nature is far above the human technology. These also serve as a stark reminder to the research and the planning communities the enormity of damages, and the repercussions of destructive earthquakes on social and economic fronts. The only way to avoid this unbearable truth is to mitigate the disasters by protecting the existing as well as new infrastructures through some innovative means, so that they are ready to face the challenges posed by natural catastrophe. Disaster mitigation measures, thus, are becoming matters of worldwide interest.

Retaining structures are integral part of any infrastructure system. They frequently represent the key elements of port and harbors, transportation system lifelines, and other infrastructural facilities. Collapse of retaining structures accompanied with disastrous physical and economic consequences are common in many historical earthquakes (Ishihara, 1997; Seed and Whitman, 1970). The predominant damage occurs in quay walls, bridge abutments, freeway structures etc. Thus, in seismically active zones frequented by strong earthquakes, adequate design of retaining structures assumes significant importance. The prerequisite for such design is, indeed, proper estimation of the seismic earth pressure through comprehensive analysis taking into consideration the soil properties, the construction conditions and the other associated factors.

Post earthquake survey of the 1995 Hyogo-ken Nanbu earthquake, Kobe (JGS/JSCE, 1996; Kamon et al, 1996) revealed the damages suffered by many waterfront retaining structures. Significant theoretical and experimental works have been done on the subject (Dickenson et al, 1998; Iai et al, 1998; Inagaki et al, 1996; Ishihara et al, 1996; Towhata et al, 1996). Many quay walls were reported to have suffered damages due to unexpected displacements (PIANC, 2001). The displacements of the quay walls during the earthquake were among the largest recorded in the history of port facilities in Japan. Maximum seaward movement of the wall recorded in that earthquake was 5 m and the maximum tilting recorded was 4 degrees towards the sea. About the same order of magnitude of settlement was induced in the soil backfill behind the walls due to the strong earthquake motion. Most of these reported damages were attributed to the two major factors; (1) soil failures due to liquefaction, subsidence of the backfill soil and liquefaction of the foundation soils beneath the caisson walls and (2) The structural failures due mainly to seaward ground movement induced by the strong inertia force. However, the effect of the increased or decreased lateral earth pressure during the earthquake prior to and/or after liquefaction cannot be entirely ruled out (Ghalandarzadeh et al, 1998; Towhata et al, 1996).

Most damages to port structures are the results of soil-structure interaction during the earthquake shaking. Therefore, seismic analysis and design should also take into the account, both the geotechnical and structural aspects of the port structures. In seismically active areas, there exist numerous structures that do not even satisfy the current design standard for earthquake resistance. Seismic performance of these structures is, therefore, sometimes at risk and questionable, as they likely to suffer excessive deformation or damages resulting from the increased earth pressures during the earthquake. Therefore, it is necessary to implement a cost-effective technique to upgrade such structures, and hence improve their seismic performance.

On the other hand, with growing concern for sustainable environment, new and promising technologies are emerging to turn industrial waste products (such as scrap tires) into a valuable resource. In many industrialized countries roughly one passenger car tire is generated per population in a year (STMC, 1997). In Japan, an estimated 102 millions scrap tires are generated annually (JATMA/JTRA, 2003). 90% of these are recycled for thermal, material and retreading purposes. However, the share of recycling as materials is still far from satisfactory. A major share of scrap tire recycling goes to the thermal recycling, a process that generates carbon dioxide (CO_2). The amount of CO_2 emission by material recycling is only 1/4 that of thermal recycling. Thermal recycling of tire chips is, thus, not environmentally friendly. Article 3 of the Kyoto Protocol ensures that the aggregate anthropogenic carbon dioxide equivalent emissions of the greenhouse gases do not exceed their assigned amounts (http://unfccc.int). Therefore, thermal recycling is contradictory to the principle of Kyoto Protocol, which entered into force since February 2005. In order to achieve a sustainable development, tire makers, scrap tire associations and construction industries in Japan are making ardent efforts to reduce the share of thermal recycling and increase the share of material recycling. In addition, the unaccounted amount of scrap tires, which constitutes the remaining 10% of the total scrap tires produced, put a burden on the environment due to illegal dumping. The void space of tires provides potential sites for rodents and mosquitoes, and thus detrimental to public health. In response to the environmental problems and the potential effect to the public health caused by countless illegal scrap tires around the globe, most industrialized countries have instigated legal guidelines addressing this issue. Such regulations have led to an increase of scrap tire recycling.

Increasing attention has recently been paid on using scrap tire derived recycled products in geotechnical engineering construction. Recycled products derived from scrap tires can be classified into two categories depending on the size of the grains: tire chips (size ranges from 12 mm to 76 mm) and tire shreds (size ranges from 76 mm to 305 mm). Both can be classified as a well-graded coarse grained geomaterials. Fig. 1 shows the tire chips made out of shredding scrap tires. Researches on engineering properties (such as shear strength, compressibility, creep, durability, permeability, etc) of this new class of geomaterials have been making steady progress for the past one decade (Ahmed, 1993; Benda, 1995; Edil and Bosscher, 1994; Hazarika et al, 2005a; Humphrey and Manion, 1992; Karmokar et al, 2005). With characteristics such as lightweight, compressible, permeable, durable and thermally insulating, this new class of geomaterial has a myriad of applications in civil engineering. This research is an attempt to contribute towards material recycling of scrap tires by utilizing tire chips as earthquake resistant geomaterials by exploiting its compressibility. In such applications, a major concern frequently posed is the adverse environmental effect due to leaching. Laboratory experimental studies have indicated no leaching of harmful substances from the tire chips (Karmokar et al, 2004). Environmental assessment study by Down et al (1996) have confirmed that use of tire chips below ground water level does not pose any environmental threat, since most of the emitted substances were found to be well below the environmental standard limit.

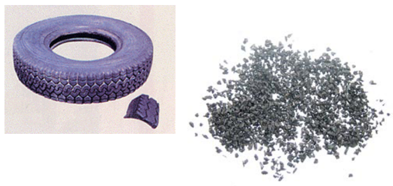


Figure 1 Scrap tire and derived product (tire chips)

As mentioned earlier, compressibility is a salient feature of tire chips. Many tend to label this characteristic of tire chips as a demerit. This particular characteristics of tire chips, however, can be utilized towards advantages rather than disadvantages, by using such material as a compressible inclusion behind the massive structures as shown in Fig. 2. Function of the sandwiched layer is to transfer only a part of the load coming to the structure, by acting as a cushion. The cushion provides flexibility, and thereby stability to a structure during the earthquake by absorbing the energy.

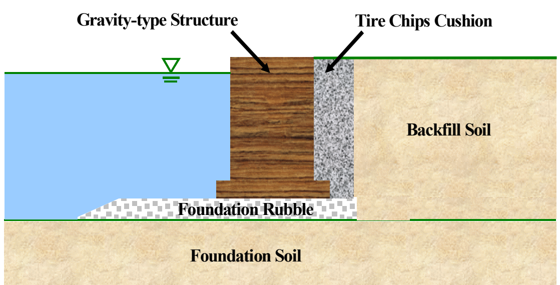


Figure 2 Compressible tire chips inclusion

The compressible inclusion technique described above, has also proven to be effective in reducing the seismic load on rigid yielding and non-yielding retaining structures (Hazarika and Okuzono, 2004b) using other compressible and lightweight materials such as EPS geofoam (Horvath, 1997). A field test (Hazarika et al, 2005b) and its numerical simulation (Hazarika, 2005) using tire chips as compressible and drainage enhancing materials have demonstrated that the static at rest pressure against a rigid non-yielding structure could be reduced substantially, thus proving the compressibility of such materials as a beneficial element in construction.

The objective of this research is to examine whether the compressible inclusion made of tire chips can reduce the load against structures during the earthquake. The shock absorbing capacity and the ductility of tire chips are also expected to aid in reducing the earthquake induced permanent displacements of the structures. The primary purpose of this research was to confirm through a model test, how the load on the structures and the permanent displacements are affected during the earthquake loading. Once the effectiveness of such soil-structure system is established, that will lead to further exploration of the possibilities of using this new class of geomaterials as an earthquake resistant reinforcing material to be reckoned on. Furthermore, this is expected to render a cost-effective design alternative for rigid non-yielding structures. As mentioned elsewhere, various material properties of tire chips have already been investigated by other researchers. Further researches are still needed to clarify the strength and compressibility characteristics under dynamic loading, and creep behavior under long time loading. Such investigations are, however, beyond the scope of the present research.

With the objectives stated above, a series of underwater shaking table tests was performed on a model caisson for verifying the seismic performance of the structure reinforced by tire chips cushion using the large underwater shaking table assembly of Structural Dynamics Division, Port and Airport Research Institute (PARI). Three different earthquake loadings were imparted to the soil-structure system and the response acceleration, the seismic load on the wall, the dynamic increment of the earth pressure acting along the wall, the residual displacement of the wall, and the water pressures at various locations of the backfill were investigated for each earthquake motion.

2. Underwater shake table testing program

At the time of the 1995 Hyogoken-Nanbu earthquake, Kobe Port had 239 quay walls, about 90% of which are caisson type quay walls (JGS/JSCE, 1996). The earthquake caused severe damage and destroyed more than 90% of the waterfront structures (Kamon et al, 1996). This has led to an increasing concern about the seismic stability of the existing and newly built port structures in Japan. In the future, two large-scale devastating earthquakes (Tokai Earthquake and Tonankai-Nankai Earthquake) are expected to strike any time in Japan. Mitigating disaster and the economic implications from these two predicted earthquakes are a major concern to planners and engineers. In order to confirm, whether the technique proposed here can be utilized as a disaster mitigation measure for the port and harbor infrastructures, an extensive testing program using underwater shaking table test was undertaken so that the soil-structure interaction behavior during the earthquake can be understood well for such waterfront structures.

2.1 Shaking table assembly

The large three dimensional underwater shaking table assembly of structural dynamics laboratory was used in the testing program. The shaking table is circular with 5.65 m in diameter and is installed on a 15 m long by 15 m wide and 2.0 m deep water pool. The detailed specifications of the shaking table assembly can be found in Iai and Sugano (2000) and Sugano et al (1996).

2.2 Model construction and instrumentations

A caisson type quay wall (model to prototype ratio of 1/10) was used in the testing. Fig. 3 shows the cross section of the soil box, the model caisson and the locations of the various measuring devices (load cells, earth pressure cells, pore water pressure cells, accelerometers and displacement gauges). The model caisson consists of three parts; the central part (width 50 cm) and two dummy parts (width 35 cm each) as shown in Fig. 4. All the monitoring devices were installed at the central caisson to eliminate the effect of sidewall friction on the measurements. Three earth pressure cells (EP1, EP2 and EP3) were attached to the caisson by flushing them against a vinyl panel glued to the steel caisson. The model caisson was made of steel plates filled with dry sand and sinker to bring its center of gravity to a stable position.

The soil box was made of a steel container 4.0 m long, 1.25 m wide and 1.5 m deep. As shown in Fig. 3, the model consists of 0.1 m of bedrock layer, 0.45 m of seabed layer of dense compacted sand (relative density = 78.59%), foundation rubble and 0.85 m high backfill. Bedrock layer was prepared using jet cement with a weight ratio of 10:3. The foundation rubble beneath the caisson was prepared using Grade 4 crushed stone with particle size of 13 mm ~ 20 mm. The backfill and the seabed layer were prepared using Sohma sand (No. 5). The end of the backfilling area in the steel container was sealed with unwoven textile to eliminate the effect of the rigid boundary. In order to achieve the plane strain conditions, the side wall of the container was made rigid.

The dense foundation sand representing the seabed layer was prepared in two layers. After preparing each layer, the whole assembly was shaken with 300 Gal of vibration starting with a frequency of 5 Hz and increasing up to 50 Hz. Backfill was also prepared in stages using free falling technique, and then compacting using a manually operated vibrator (capacity 350W, frequency 191~217 Hz, diameter ϕ=32 mm, and length L=260 mm).

After constructing the foundation and the backfill, and setting up of the devices, the pool was filled with water gradually elevating the water depth to 1.3 m to saturate the backfill. This submerged condition was maintained for two days so that the backfill attains a complete saturation stage. The initial ground surface heights were measured at 18 different target points spread over the entire backfill. They were also subsequently measured at the end of each oscillation to monitor the earthquake induced ground surface settlement.

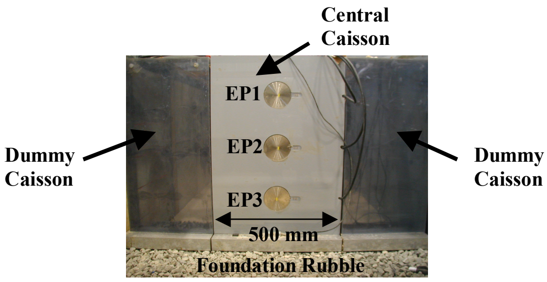


Figure 4 Model caisson with earth pressure cells

2.3 Test cases

As shown in Fig. 5, two series of tests were conducted. In one series (Case A), a caisson with a conventional sandy backfill was used. In another series (Case B), behind the caisson, a 30 cm thick layer of tire chips (average grain size 20 mm) was placed vertically down as buffer cushion. The thickness chosen here was 0.4 times of the wall height. In actual practice, the design thickness will depend upon a lot of other factors such as height and rigidity of the structure, compressibility and stiffness of the cushion material. In compressible inclusion

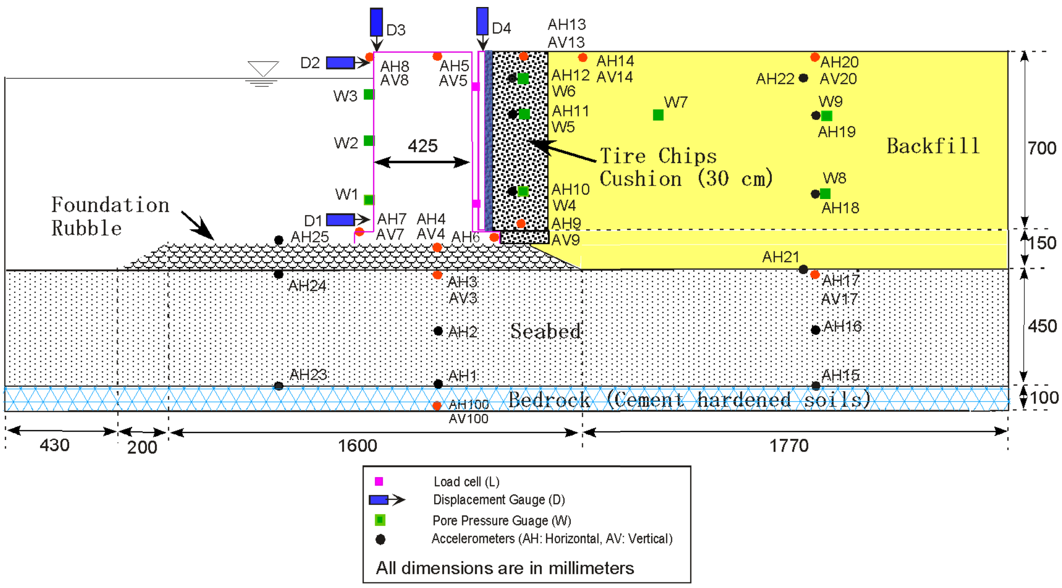
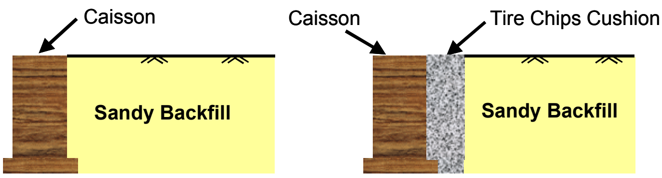


Figure 3 Cross section of the test model

applications, there seems to be an optimum value for the cushion thickness, beyond which an increase in thickness will not lead to a proportionate decrease of the load (Hazarika et al, 2002). However, for simplicity, only a constant thickness cushion was used.



(a) Case A (b) Case B

Figure 5 Test cases

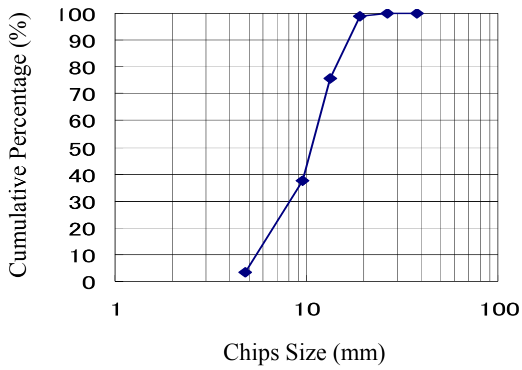


Figure 6 Grain size distribution curve for tire chips

2.4 Test materials and test methods

The grain size distribution of the tire chips used in the testing is shown in Fig. 6. The cushion layer was prepared by filling tire chips inside a bag made from geotextile product. Geotextiles are required to wrap the tire chips so that they do not mix with the surrounding soils. Such confinement also makes the execution of backfilling easier. Furthermore, the presence of geotextiles also prevents flowing of sand particles into the chips structure, and thus prevents clogging and mixing, which may affect the compressibility and permeability of the chips. The average density of the tire chips achieved after filling and tamping was 0.675 t/m^3. Fig. 7 shows the resulting cushion layer behind the caisson.

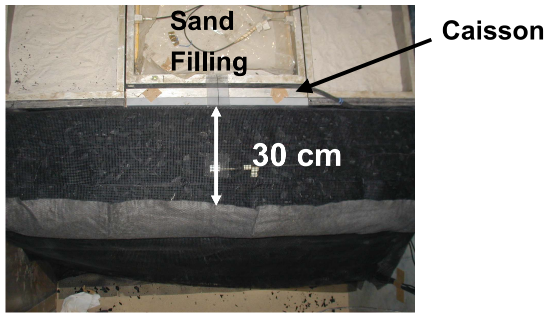


Figure 7 Tire chips cushion behind the caisson

The grain size distribution of the Sohma sand, which was used as the foundation and the backfill soil, are shown in Fig. 8 for two relative densities. The densities of the foundation and backfill soils in each cases are tabulated in the Table 1. The relative densities of the backfill sand were calculated using the maximum and minimum void ratios obtained for the sand from the drained triaxial test and the obtained dry densities for each test case. The relative densities thus calculated were found to be 45.94% and 49.92% for the Case A and Case B respectively. This implies that the backfill soil is liquefiable. Since liquefaction tends to increase the earth pressure, the presence of tire chips cushion is expected to protect the structure from the adverse effect of liquefaction during the earthquake. Liquefiable backfill was thus selected on purpose. On the other hand, the sea bed relative density was calculated to be 78.59 %, implying a non-liquefiable foundation deposit.

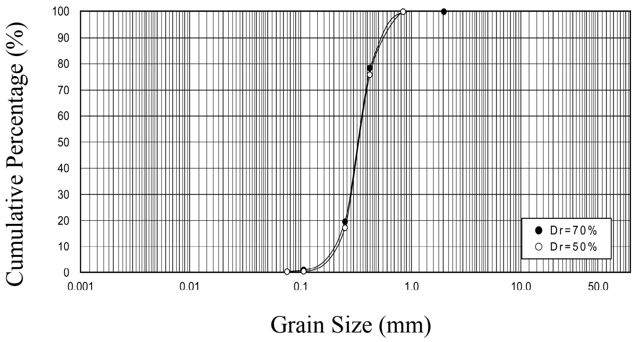


Figure 8 Grain size distribution curve for Sohma sand

Shaking table test requires special consideration to define appropriate scaling law between the prototype and the model. The similitude of various parameters in 1g gravitational field (Iai, 1989) for the soil-structure-fluid system adopted in this study are shown in Table 2 for a model to prototype ratio of 1/10. It is worthwhile mentioning here that, the material particles size and compressibility of the material are assumed to remain unchanged, for the model and the prototype.

From the start of the backfilling to the end of the water filling, the variation of the static earth pressures were measured and are shown in Figs. 9a and 9b for the two test conditions (Case A and Case B). The final pressures indicated in the figures also include the static water pressures. The initial static earth pressure can be obtained by subtracting the water pressure at the respective depths. Comparison of the results in Figs. 9a and 9b indicate that the static earth pressure against the structure is also reduced by the use of the compressible material such as tire chips. The effect of such compressible inclusion technique using some other artificial geomaterials has been experimentally proved (Hazarika et al, 2002; Hazarika and Sugano, 2004; Tsukamoto et al, 2002).

Table 1. Densities of foundation sand and backfill sand

Test Cases	Seabed (t/m^3)	Rubble Mound (t/m^3)	Backfill (t/m^3)
Case A	1.502	1.83	1.386
Case B	1.502	1.78	1.398

Table 2. Similitude for 1g field

Items	Prototype／Model	Scale factor
Length	λ	10
Time	$\lambda^{0.75}$	5.62
Density	1	1
Stress	λ	10
Water Pressure	λ	10
Displacement	$\lambda^{1.5}$	31.62
Acceleration	1	1

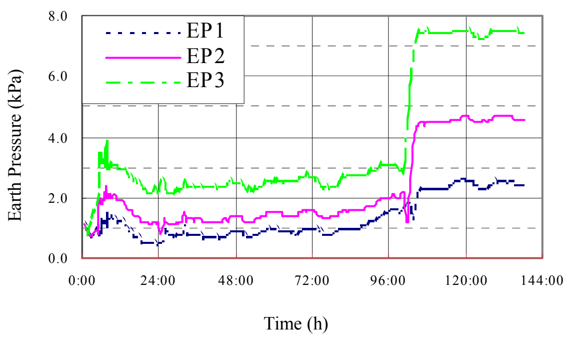


Figure 9a Time variation of static earth pressure (Case A)

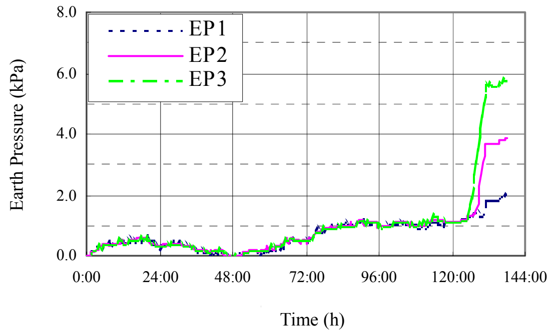
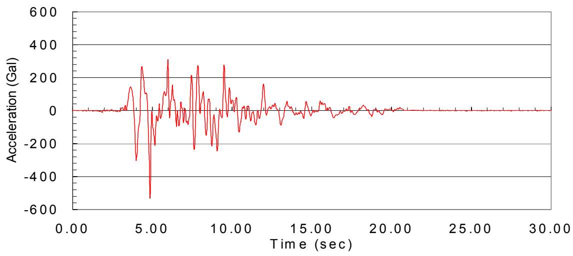


Figure 9b Time variation of static earth pressure (Case B)

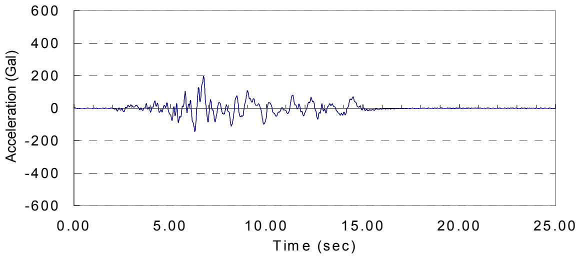
2.5 Input earthquake motions

Three different earthquake loadings were imparted to the soil-structure system during the tests. The input motions selected were: (1) the N-S component of the strong motion acceleration record at the Port Island, Kobe during the 1995 Hyogo-ken Nanbu earthquake (M 7.2), (2) the N-S component of the earthquake motion recorded at the Hachinohe port, during the 1968 Tokachi-Oki earthquake (M 7.9), and (3) a scenario earthquake motion created artificially assuming an earthquake that is presumed to occur in the southern Kanto region with its epicenter at Ohta ward, Tokyo. The wave records of these input motions are shown in Fig. 10. The third motion (Fig. 10c) is characterized by long duration and low frequency in contrast to the 1995 Kobe earthquake, which had a small duration. Durations of the shaking in the model testing were based on the time axes of these accelerograms, which were reduced by a factor of 5.62 according to the similitude shown in Table 2.

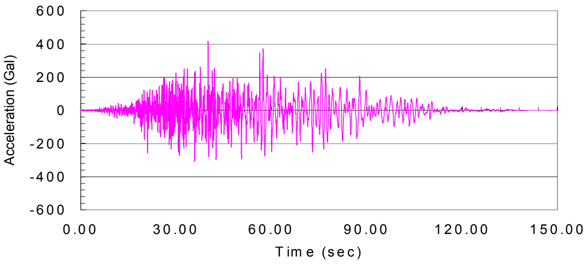
For a given seismic excitation, the response accelerations in various parts of the structure and the backfill were monitored. The seismic load on the wall, the dynamic increment of the earth pressure acting along the wall and the water pressures in various locations of the backfill were measured during the excitation. Since grain size of tire chips are comparatively large (average grain size = 20 mm), the earth pressure cells instrumented in the caisson were of larger size diameter (ϕ=100mm) in order to avoid instability and unreliability of the measured data (Miura et al, 2003).



(a) Port Island, Kobe (Hyogo-ken Nanbu earthquake)



(b) Hachinohe (Tokachi-Oki earthquake)



(c) Ohta ku, Tokyo (Simulated Southern Kanto earthquake)

Figure 10 Input strong motion records

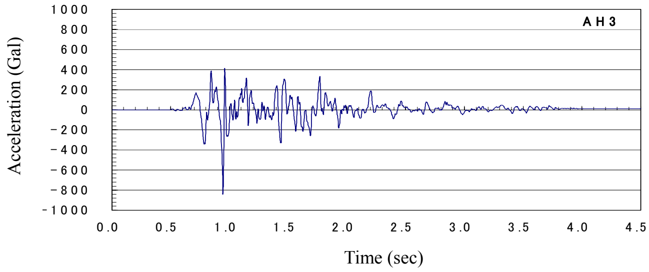
3. Test results and analyses

In this section discussions are focused only for the test series conducted using the Port Island, Kobe earthquake motion (Fig. 10a). Subsequent discussions in Section 4 will give details about the behavior of the soil-structure system under the other earthquake motions.

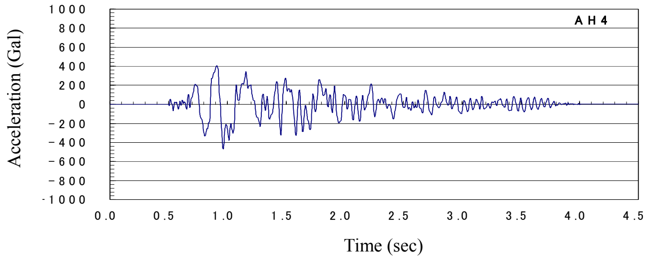
3.1 Response acceleration

The horizontal accelerations at various locations (AH3, AH4, AH5 and AH6 in Fig. 3) are shown in Fig. 11 for the test Case A. Fig. 12 shows the same for the test Case B. Fig. 13 compares the acceleration in the backfill at a location close to the caisson (AH10) for the two cases.

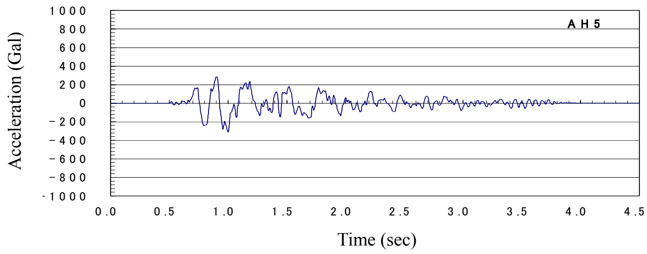
Fig. 14 compares the acceleration in the backfill at a location away from the caisson, but at the same vertical height (AH18 in Fig. 3) for the two test cases. These figures reveal that the response accelerations within the tire chips, backfill and the caisson differ depending on the absence (Case A) or the presence (Case B) of the buffer cushion.



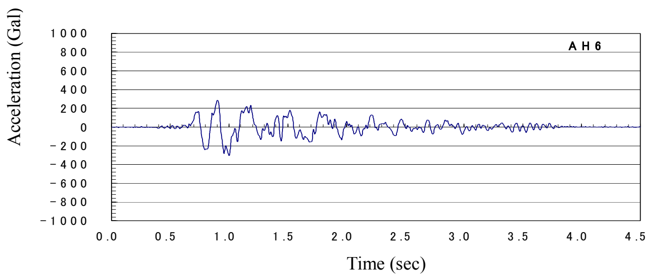
(a) At the seabed



(b) At the rubble mound

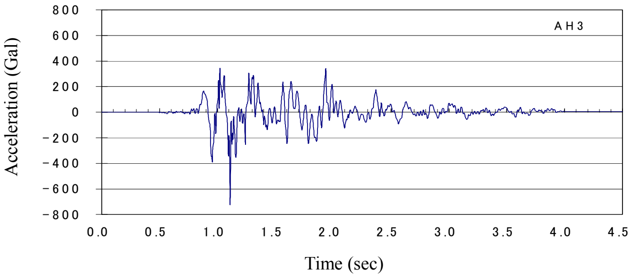


(c) At the caisson top

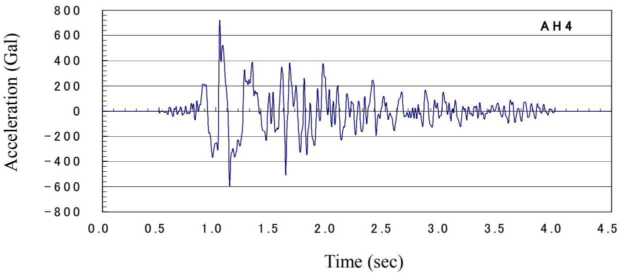


(d) At the caisson bottom

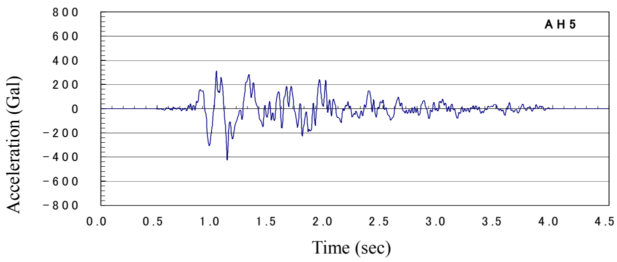
Figure 11 Response accelerations at various locations (Case A: for sandy backfill)



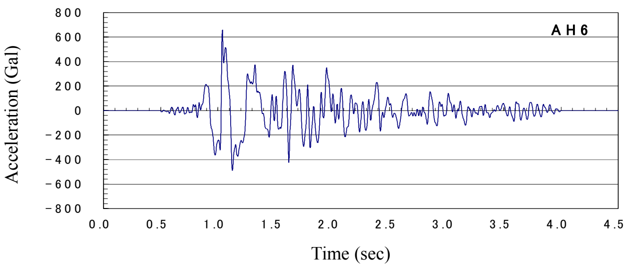
(a) At the seabed



(b) At the rubble mound



(c) At the caisson top



(d) At the caisson bottom

Figure 12 Response accelerations at various locations (Case B: for backfill with cushion)

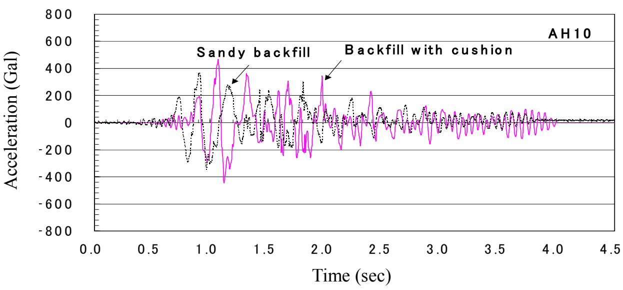


Figure 13 Comparison of response accelerations (AH10)

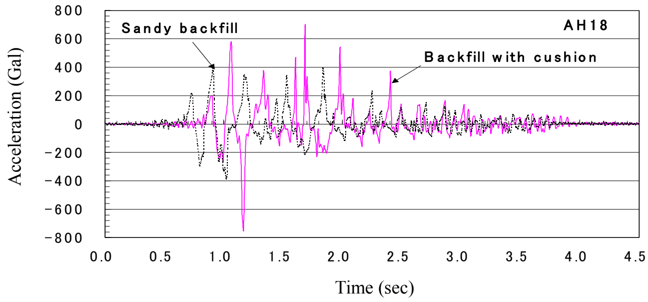
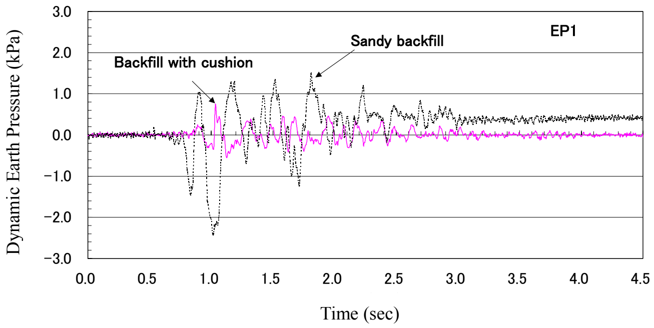


Figure 14 Comparison of response accelerations (AH18)

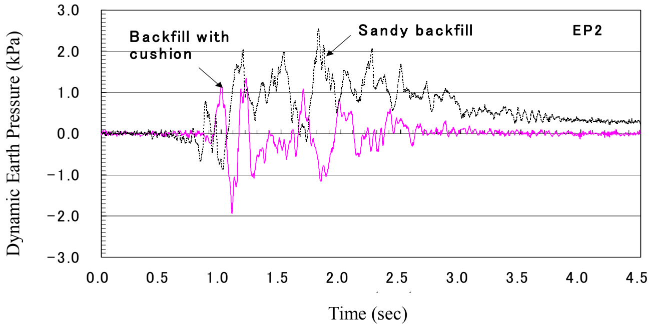
3.2 Seismic earth pressure

Figs.15(a)~(c) show the time histories of the measured horizontal seismic thrust acting on the top, the middle and the bottom of the caisson for both the test cases. It can be observed that, as compared to conventional backfill, the use of tire chips cushion yield a significant reduction of the seismic earth pressure acting on the caisson at each depth. While the caisson without any protective cushion experiences high fluctuation of the earth pressure with a predominant peak, the earth pressure on the cushion-protected caisson stabilizes soon. The maximum amplitude of the latter is also much lower than that of the former. This implies that the seismic performance of the caisson improves with the use of the sandwiched cushion.

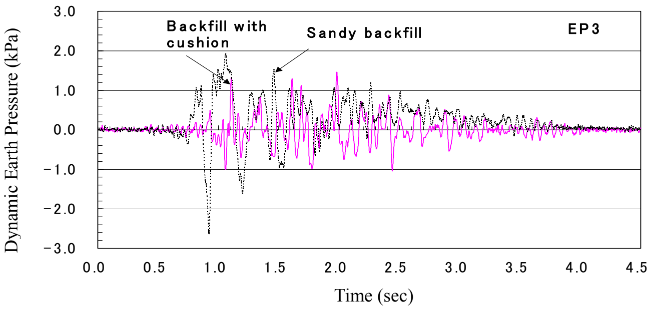
One interesting point to be noted here is that, the incremental earth pressures are acting in the opposite direction of the inertia force in the case of backfill with cushion (Case B). Such phase differences were not observed in Case A. Another interesting observation is that, for the sandy backfill condition, the pressure does not come to a stabilized state immediately even at the end of the load application (3.0 sec). On the contrary, the cushion sandwiched backfill comes to the stabilized state immediately upon stopping the load. This may be due to the residual earth pressure, which takes time to come to the original state even after ceasing of the seismic load. The residual pressure generated is the result of the excess pore water pressure, which takes time to stabilize in Case A. A subsequent discussion in Sub-section 3.7 on liquefaction behavior of the backfill will elaborate on this topic.



(a) At the top most cell



(b) At the middle cell



(c) At the bottom most cell

Figure 15 Time histories of the seismic earth pressure on the caisson

3.3 Distribution of the seismic thrust

At a particular time (t=2.39 sec) of the time history, the distribution of the seismic increment was plotted as shown in Fig. 16. A substantial reduction of the seismic increment was achieved, particularly in the middle of the caisson. In Fig. 17, the total seismic thrusts acting on the caisson quay wall are plotted. The total seismic force was obtained by adding the total static force (minus the static water pressure in Figs. 9a and 9b) to the seismic incremental thrust obtained in Fig. 16. It can be observed that the total seismic force on the wall could be reduced to almost half in this particular case. Reduction of the seismic thrust implies a lower design load, which implies a smaller caisson width, which in return will lead to a low material cost. Thus, the technique developed here can lead to a cost effective design not only in terms the backfill material, but also in terms of the structural material as well. However, the test results give only a qualitative pictures of the load reduction capacity of the developed technique. For any quantitative evaluation of the load reduction capabilities, the effect of other related factors such as cushion thickness as well as cushion shape, structural type etc need to be examined in detail.

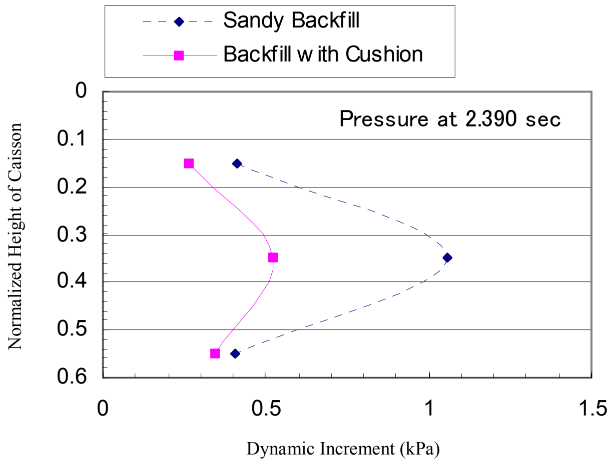


Figure 16 Incremental earth pressure

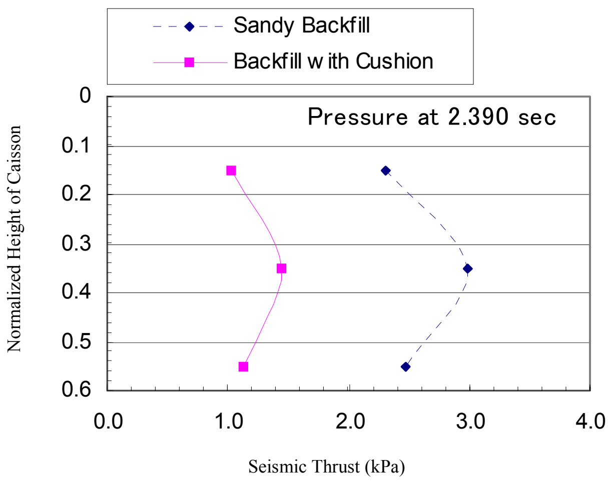
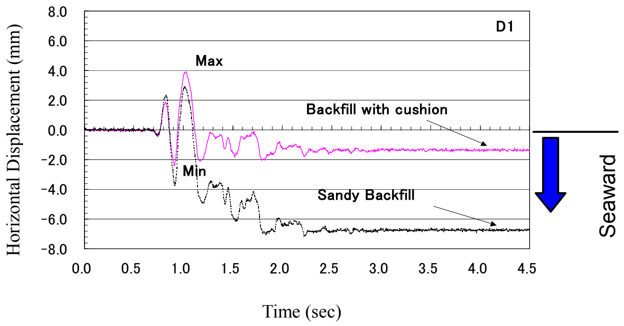


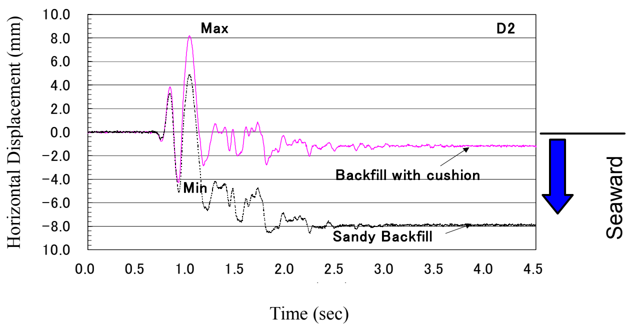
Figure 17 Total seismic thrust on the caisson

3.4 Horizontal displacement of the caisson

In actual practice, it is of greatest importance whether the permanent structural deformation will lead to halting of the port operation in the event of a destructive earthquake. Even if the seismic earth pressures are reduced, excessive deformations of the structures (like the ones during the 1995 Kobe earthquake) can be very detrimental. In order to see, whether the developed method can minimize the maximum horizontal displacement as well as the residual horizontal displacement experienced by the caisson, the time histories of the horizontal displacements (D1 and D2 in Fig. 3) during the earthquake loading for the two cases are compared in Fig. 18. In this figure, the negative displacement indicates a seaward displacement of the caisson.



(a) Bottom (at 5 cm from the caisson toe)



(b) Top (at 5 cm from the caisson top)

Figure 18 Time histories of the horizontal displacements

The figures reveal that the maximum displacement experienced by the caisson with tire chips cushion is toward the backfill in contrast to the caisson without cushion, in which case it is seaward. The compressibility characteristics of the tire chips renders flexibility to the soil-structure system, which allows the caisson to bounce back under its inertia force, and this tendency ultimately (at the end of the loading cycles) aids in preventing its excessive seaward deformation. However, the caisson without any protective cushion experiences a very high seaward displacements from the beginning due to its inertia. As a consequence, the caisson can not move back to the opposite side and ultimately suffers from a huge seaward displacement. The less displacement magnitudes experienced by the cushion protected structure are not solely due to the compressibility of the tire chips, but also the result of strong earthquake resistant characteristics of the tire chips.

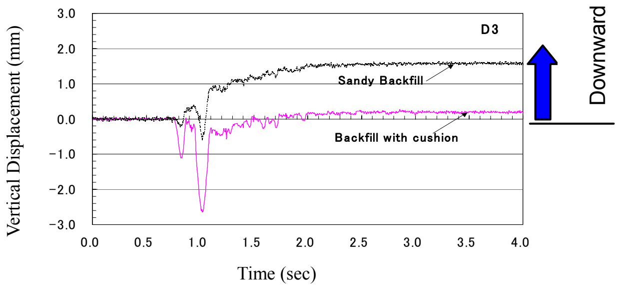
A quantitative observation of these figures also indicate that the unprotected caisson experiences about 7.87 mm and 6.71 mm of horizontal residual displacement at the top and the bottom respectively. This implies an about 24.88 cm horizontal residual displacement at the top for the prototype calculated using the scaling factor shown in Table 2. However, the horizontal residual displacement experienced by the caisson with tire chips cushion are a bare minimum with 1.39 mm at the top and 1.17 mm at the bottom respectively. This implies an about 4.39 cm for the prototype (cushion thickness 3.0 m), which is only 1/7 of the displacement for the unprotected caisson.

3.5 Vertical displacement of the caisson

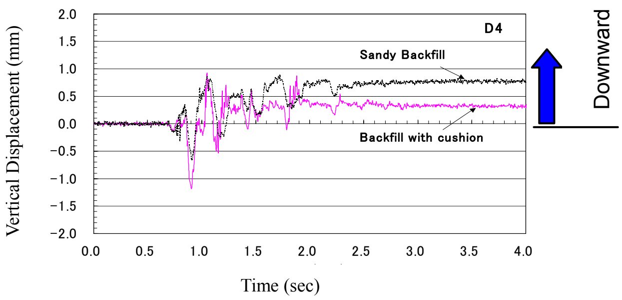
The time histories of the vertical displacements of the caisson (D3 and D4 in Figure 3) during the earthquake loading for the two cases are compared in Figs. 19(a) and (b). In these figures, a positive value implies a downward movement of the caisson. It can be observed that, even though the maximum displacement during the loading sometimes exceeded that of unprotected caisson, the residual vertical displacement remains significantly less for the cushion protected caisson (a mere 0.208 mm). For the unprotected caisson it has reached a value as high as 1.58 mm (7 times of the former).

3.6 Rocking characteristics

The time history of the rocking characteristics of the caisson is compared in Fig. 20. It is interesting to note that the caisson with tire chips cushion experiences maximum rocking towards the backfill, while the caisson without cushion experiences towards the sea. In cushion protected caisson, due to the highly compressible and ductile properties of the tire chips, the caisson can come back to its original position, bringing stability to the system soon after a few cycles of excitation. However, for the unprotected caisson, the rocking induces a seaward permanent displacement to the structure. Thus, it can be inferred that, by utilizing the proposed technique, the rocking motion of a structure can be substantially reduced if not prevented. Reduction and/or prevention of the rocking imply an increased stability to a structure during the earthquake loading contributing to the enhancement of its seismic performance.



(a) Displacement at the seaside caisson edge



(b) Displacement at the backfill side caisson edge

Figure 19 Time histories of the vertical displacements

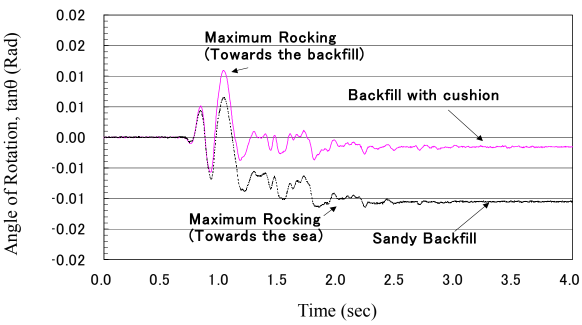


Figure 20 Time histories of the tilting angle

3.7 Liquefaction potential of the backfill

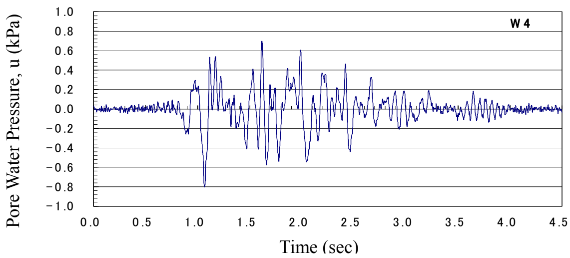
In order to observe the shear deformation behavior of the backfill during earthquake, which may lead to the liquefaction, the excess pore water pressure measured by the installed water pressure gauges are plotted in Figs. 21~23.

The onset of liquefaction in the sandy backfill can be determined by a parameter called the pore water pressure ratio (r_u), which is defined as follows:

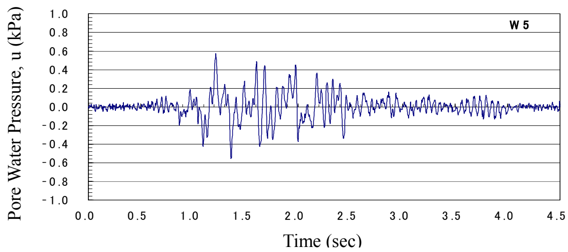
$$r_u = \frac{u}{\sigma'_{v0}} \tag{1}$$

Here, u is the excess pore water pressure and σ'_{v0} is the initial vertical effective stress at the particular depth for a horizontally layered ground.

Figs. 21(a) and (b) show the time history of the excess pore water pressure (u) recorded by the gauges (W4 and W5) installed inside the tire chips cushion (refer to Fig. 3). It can be seen that the highly granular tire chips layer prevents any development of such pressure, except for the little increase in the form of dynamic water pressure.



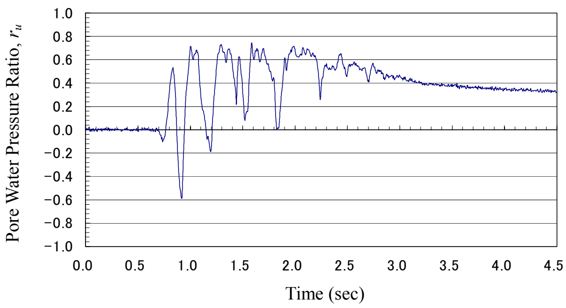
(a) At the bottom one third location



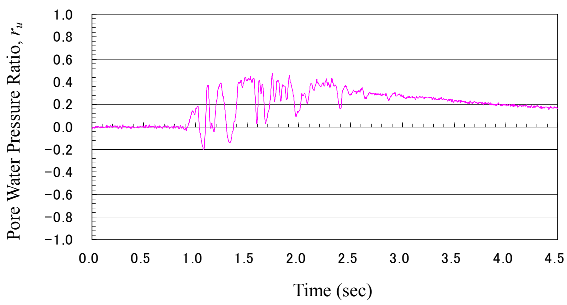
(b) At the top one third location

Figure 21 Time histories of the excess pore water pressure

Figs. 22(a)~(b) show the time history of the excess pore water pressure ratio (r_u) developed at a location 0.65 m from the caisson (at the depth of 0.25 m). The unprotected caisson (Fig. 22a) backfill shows a significant development of the excess pore water pressure (twice that of the cushion protected caisson) and thus may experience liquefaction-induced failure. The cushion protected caisson backfill (Fig. 22b), on the other hand, does not experience appreciable increase of the pore water pressure, and thus, is not likely to undergo liquefaction. The dissipation of the pore water pressure is also faster in this case. One reason for this is the presence of high permeability (1.5 to 2.5 cm/sec depending on the void ratio) materials like tire chips in the vicinity. Granular and highly permeable tire chips give the pore water pressure a chance to dissipate and consequently, there is a less chance of the increase of pore water pressure in the vicinity of the caisson. Compressibility of the tire chips also plays its role here. The presence of highly compressible tire chips cushion can control the shear yielding of the sand particles, and thus increases the cyclic mobility of the backfill soil. Comparison of these two figures, also indicate an early onset of the rise of pore water pressure for the unprotected caisson than the cushion protected caisson.



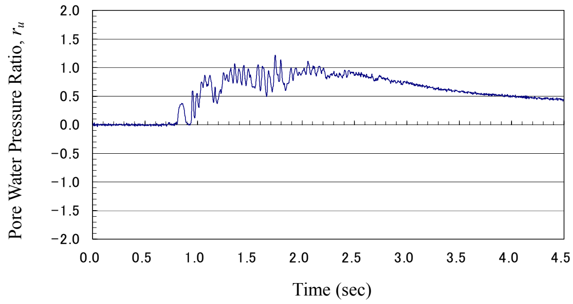
(a) Case A (Sandy backfill)



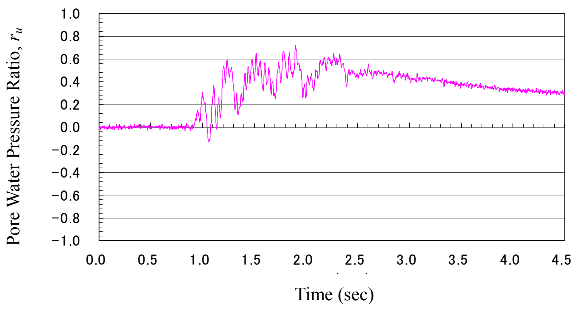
(b) Case B (Backfill with cushion)

Figure 22 Excess pore water pressures at location W7

Figs. 23(a)~(b) show the time history of the excess pore water pressure ratio at a location 1.25 m away from the caisson. In this case, the pore water pressure develops appreciably in both the cases, which may lead to the onset of liquefaction in the backfill. There is also no appreciable delay in the generation of the pore water pressures. Thus, it can be concluded that, even though in the vicinity of the cushion the tire chips helps dissipating the excess pore water pressure generated during the earthquake loading, beyond the influence zone (which may vary depending on the cushion thickness and the relative density of the backfill) of the tire chips cushion, there is likelihood of liquefaction unless some protective measures are taken to prevent such occurrences.



(a) Case A (Sandy backfill)

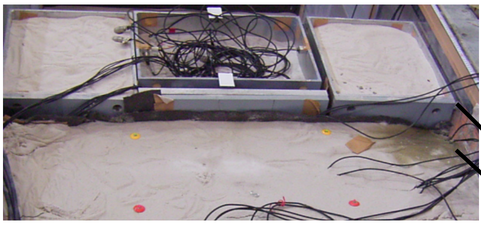


(b) Case B (Backfill with cushion)

Figure 23 Excess pore water pressures at location W9

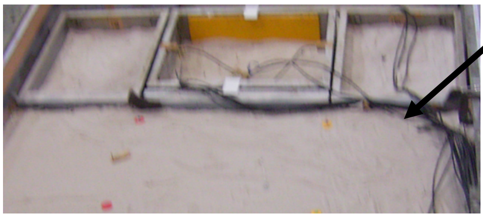
3.8 Ground surface settlement

Analyses of the test results discussed so far have indicated that the developed technique can render stability to structures by reducing the seismic increment of the load. However, in order to examine, whether the developed technique can contribute towards the safe operation of the port function after a devastating earthquake, the differential settlements in the backfill with and without cushion need to be compared. In order to observe such responses, a more severe earthquake loading was imparted to the soil-structure system, by subjecting it to acceleration amplitude of 1.5 times that of the Kobe earthquake (Fig. 10b). Fig. 24 compares the state of backfill settlement at the end of the loading for the Case A and the Case B. It can be observed that, while the structure with conventional sandy backfill experiences a very severe differential settlement (a prototype equivalent of more than 3.162 m), the structure with cushion does not undergo appreciable differential settlement.



>10 cm
(3.162 m in prototype)

(a) Case A (Sandy backfill)



No significant differential settlement

(b) Case B (Backfill with cushion)

Figure 24 Ground surface settlements

4. Effect of earthquake motions

Earthquake excitation consists of various types of half cycles of waves ranging from large to small amplitudes, long to short duration, high to low frequency. As mentioned before, three different earthquake motions (Fig. 10) were applied in the testing program. All these three motions are characterized by different frequency, amplitude and duration.

The Port Island wave record (named hereafter PI) of the 1995 Hyogoken Nanbu earthquake was adopted because of the extensive damages brought by that earthquake to caisson type quay walls. An estimated 186 caisson type quay walls were damaged during the earthquake (JGS/JSCE, 1996). The Hachinohe port wave record during the 1968 Tokachi Oki earthquake (named hereafter HN) was used as another input motion because it is the de facto ground motion in the Japanese design standard of the Port and Airport structures. On the other hand, the Ohta ku scenario earthquake motion was selected because it is a Level 2 earthquake motion (named hereafter L2). The lessons learned from the devastating earthquakes in the 1990s have given birth to an emerging design methodology called the performance-based design (SEAOC, 1995; Steedman, 1998). In performance-based design, dual approach using the Level 1 and Level 2 earthquake motions are adopted (JSCE, 2001; PIANC, 2001).

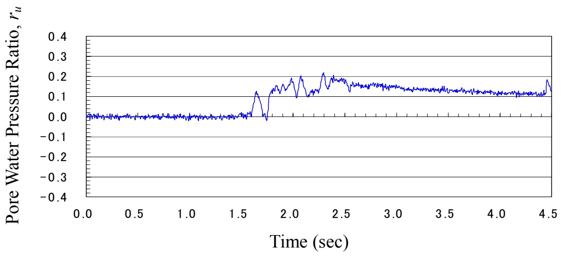
The purpose of using the three different earthquake motions described above was to investigate, how the developed technique performs depending on the earthquake intensity and severity. The differences of the earthquake motions and their characteristics also reflect the difference in the site conditions. Nozu (2004) emphasized the importance of adopting a design ground motion reflecting the site characteristics.

4.1 Onset and prevention of liquefaction

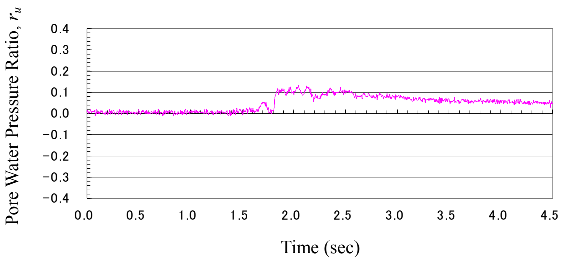
Figs. 25~27 compares the behavior of the pore water pressure generation at a location 0.65m from the caisson (location W7 in Fig. 3) under the three different earthquake motions (PI, HN and L2 records) for the two test cases. Observations indicate that at the same location, for Case A, the HN wave generated the least pore water pressure ratio (maximum=0.2), followed by the L2 wave (maximum=0.6). The PI wave generated the maximum pore water pressure ratio of 0.7. Therefore, as far as the present test conditions (type of soils, backfill soil density, tire chips cushion thickness) are concerned, it can be said that while the HN type wave may not induce liquefaction at that location, the PI wave and L2 wave is more likely to cause liquefaction. Similar observations for the Case B have indicated that the pore water pressure ratio is 0.2, 0.4 and 0.5 respectively for the HN, PI and L2 records respectively, indicating no likelihood of liquefaction.

The above test results indicate that the question of liquefaction does not arise at all at these locations for the backfill with the tire chips cushion if the pore water pressure build-up is used for defining the onset of liquefaction. However, in practice, liquefaction is characterized in two ways (PIANC, 2001). Differences may arise regarding the liquefaction behavior depending on the definition, especially if soil characteristics are significantly different from those in the field.

An interesting observation, here, is regarding the rate of dissipation of the pore water pressure. The rate is the highest for the L2 wave and the lowest for the PI wave. These observations lead to a conclusion that the type of the strong motion wave can have an influence on the liquefaction behavior of the backfill soils. This particular investigation has indicated that the HN type of wave is less likely to cause liquefaction related damage. However, substantial evidences are required taking into the account the backfill soil properties of the prototype to confirm this fact and that is beyond the scope of this research.

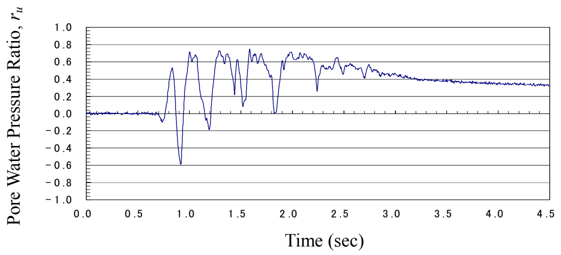


(a) Case A (Location: W7)

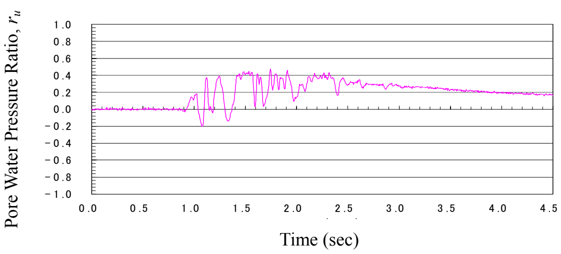


(b) Case B (Location: W7)

Figure 25 Liquefaction potential of under HN record

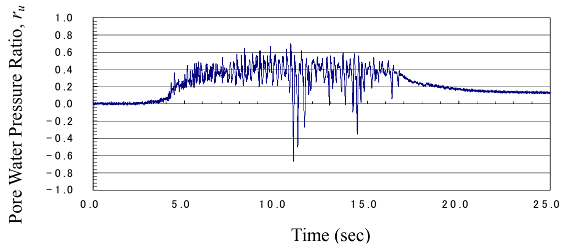


(a) Case A (Location: W7)

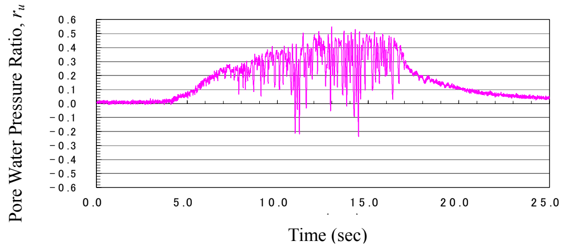


(b) Case B (Location W7)

Figure 26 Liquefaction potential under PI record



(a) Case A (Location: W7)



(b) Case B (Location: W7)

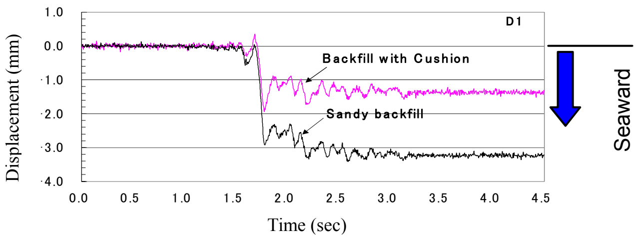
Figure 27 Liquefaction potential under L2 record

4.2 Horizontal displacements of the caisson

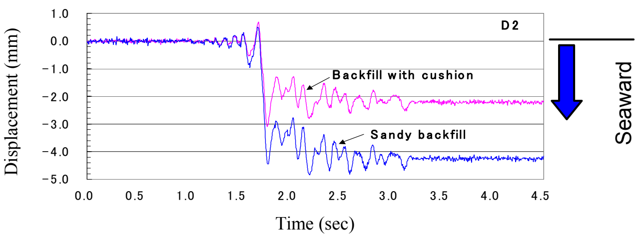
Figs. 28 and 29 show the time history of the residual seaward displacement (D1 and D2) of the caisson for the HN and L2 type records respectively. The figures reveal that the L2 type earthquake yields higher residual displacement than the HN earthquake. In fact, it is the highest among the three motions (Refer to Fig. 18, section 3 for the PI record). Another important observation is that unlike the PI wave and the HN wave, the L2 wave does not yield a positive displacement (towards the backfill). The caisson starts to move seaward from the beginning of the oscillations. That perhaps the reason for an unexpectedly higher residual displacements (6.93 mm at the bottom and 7.84 at the top) even for the cushion protected caisson for this type of wave record.

4.3 Residual displacements of the caisson

The seaward permanent horizontal displacements of the caisson under the three earthquakes motions (PI, HN and L2) are compared in Fig. 30 for the Case A and the Case B. This figure reveals further that, the cushion protected caisson experiences less residual displacements than the unprotected caisson during an earthquake. However, in the case of the L2 type motion, even though the residual displacement itself is less compared to the unprotected case, the magnitude is relatively high as compared to the other two ground motions. This implies that for such high intensity and long duration motion, the adopted cushion thickness (30 cm) may not be adequate. It may be necessary to increase the cushion thickness to compensate for such higher deformation. The thickness, however, will not solely dictate such compensations. The thickness is a necessary, but not the sufficient factor. As discussed elsewhere, the L2 type motion is more likely to cause liquefaction in some parts of the backfill. In those locations, some preventive measures against liquefaction may be required. Further research on the soil-structure interaction under this particular earthquake record is required to arrive at a logical proposition for the thickness.

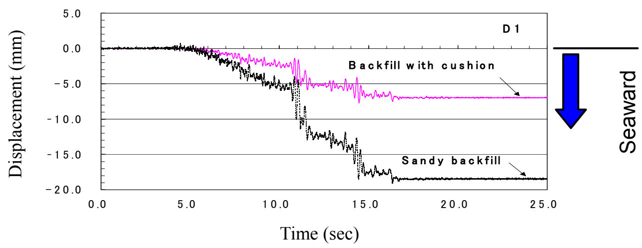


(a) Bottom (at 5 cm from the caisson toe)

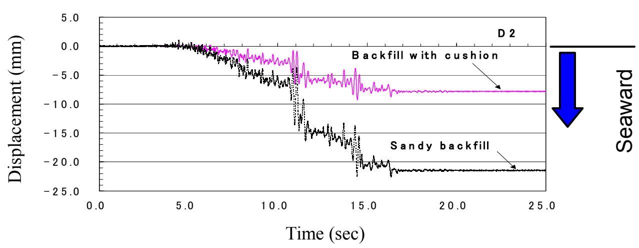


(b) Top (at 5 cm from the caisson top)

Figure 28 Displacements of the caisson under HN record



(a) Bottom (at 5 cm from the caisson toe)



(b) Top (at 5 cm from the caisson toe)

Figure 29 Displacements of the caisson under L2 motion

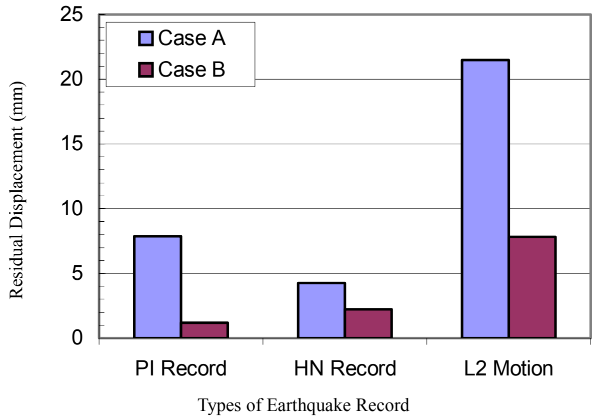


Figure 30 Comparisons of the permanent displacements of the caisson in different earthquake conditions

5. Modeling of the soil-structure interaction system

When a cushion of different stiffness characteristic is sandwiched between a structure and the backfill soil, a hybrid system of interaction is generated. In such a system, the sandwiched element possessing different stiffness and compressible characteristic yields two different interfaces: structure-cushion, cushion-soil. Proper modeling of these interfaces determines the final outcome of such soil-structure interaction analysis.

In this section, an interface model was developed for analyzing a system comprising of the backfill soil, the sandwiched cushion and the structure. The developed model is an extension of the model developed by Hazarika & Okuzono (2004) for a similar system but of different geomaterials. In conventional analysis of retaining structures, it is common to use interface element of zero thickness (Day & Potts, 1998; Hazarika & Matsuzawa, 1996) to simulate the interface. However, in a situation depicted in Fig. 2, where sandwiched material is involved, the interfaces are not exactly in a planar surface. Especially in a situation where highly granular and irregular sized tire chips particles is involved, inescapable gap exists between the interactive media, where smaller material particles can go in, producing a *thin layer interface* that participates in the overall interaction process. The idea of including the finite thickness interface developed by Desai et al (1984) is, therefore, better suited for modeling such an interactive system.

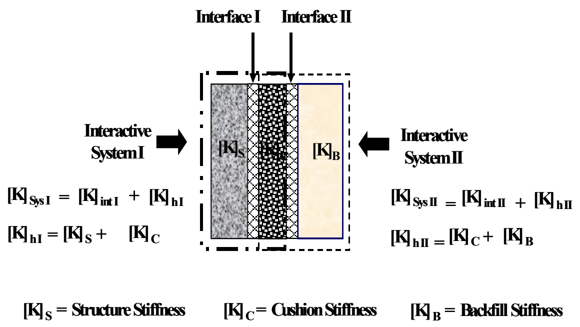


Figure 31 Modeling concepts

Fig. 31 shows the conceptual representation of the interface, which consists of two interactive systems, represented as interface system I and interface system II. The corresponding stiffness of the systems are given by the following.

$$[K]_{Sys\,i} = [K]_{Int\,i} + [K]_{h\,i} \qquad (\mathrm{i} = \mathrm{I}, \mathrm{II}) \qquad (2)$$

Where, $[K]_{hi}$ represents the sum of the stiffness of the solid elements of the participating media at the respective interfaces and are given by the following equations.

$$[K]_{h\,I} = [K]_S + [K]_C \qquad (3)$$

$$[K]_{h\,II} = [K]_C + [K]_B \qquad (4)$$

Here, $[K]_S$, $[K]_C$ and $[K]_B$ are the stiffness of structure, cushion and backfill soil respectively. In Eq. (2), the stiffness of the interfaces ($[K]_{Int\,\mathrm{i}}$ $(\mathrm{i} = \mathrm{I}, \mathrm{II})$) are given by the following equations.

$$[K]_{Int\,i} = [\overline{K}_n]_{Int\,i} + [K_s]_{Int\,i} \qquad (5)$$

In the above equations $[K_s]_{Int\,i}$ are the shear components of the interface stiffness. They can be obtained from the direct shear tests or similar interface testing. The normal behavior of the thin interfaces can be expressed by equations of the form given below.

$$[\overline{K}_n]_{Int\,I} = \alpha_1 [K_n]_{Int\,I} + \alpha_2 [K_n]_C + \alpha_3 [K_n]_S \qquad (6)$$

$$[\overline{K}_n]_{Int\,II} = \beta_1 [K_n]_{Int\,II} + \beta_2 [K_n]_C + \beta_3 [K_n]_B \qquad (7)$$

In Eqs. (6) and (7), $[K_n]_{Int\,I}$ and $[K_n]_{Int\,II}$ are the stiffness matrices of the normal direction of the Interface I and Interface II respectively. $[K_n]_S$, $[K_n]_C$ and $[K_n]_B$ represents the stiffness of structure, cushion and backfill in the normal directions respectively. The terms α_1, α_2, α_3, β_1, β_2 and β_3 are the constants called the *participation factors*. These factors represent the contribution of the respective structural material and geomaterials participating in the interactions at each interface. They, therefore, satisfy the following relationship.

$$\alpha_1 + \alpha_2 + \alpha_3 = \beta_1 + \beta_2 + \beta_3 = 1.0 \qquad (8)$$

At each interface, the sandwiched element has a finite zone of influence (Hazarika and Okuzono, 2004) within which it interacts. The influence zone was assumed to be half the thickness of the adjoining interface elements. When no cushion layer is present (which represents a conventional soil-structure interaction problem), the stiffness, $[K]_{Sys}$ for the single interactive system can be derived using Eq. (2) as follows.

$$[K]_{Sys} = [K]_{Int} + [K]_S + [K]_B \qquad (9)$$

6. Numerical analyses

Performance observation has shown that for quay wall, the effective stress analysis takes the priority and renders better results (Iai, 1998; Iai et al, 1998). However, as a first step towards implementing the developed earthquake resistant technique using a new class of geomaterials like tire chips, only a very ideal case (comprising of dry backfill as well as foundation sands) was considered here.

6.1 Simulated model

A plane strain FEM discretization of the model caisson, which was selected for the numerical experiment, is shown in Fig. 32. The model wall was 7 m in height with sandy backfill, which was assumed to be dry. The caisson rests on foundation rubble made up of ballast overlying a dense sandy layer with characteristics same as that of the backfill. As shown in Fig. 32, reflected boundaries (roller support) are allowed to have movement only in vertical directions, while fixed boundaries are restrained against both the movements. Viscous dampers were introduced at the reflected boundaries.

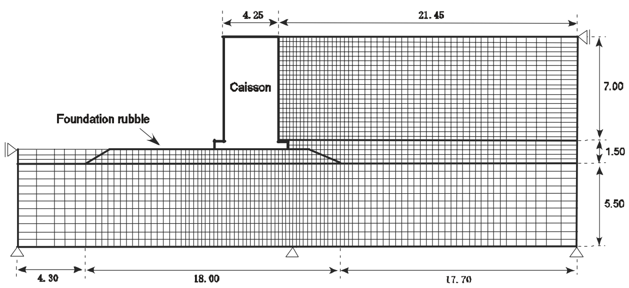


Figure 32 FEM model (all dimensions are in meters)

6.2 Material constitutive model

A 3 m thick tire chips cushion was placed behind the caisson of Fig. 32. Such hybrid type of soil-structure interaction problem needs to be analysed using separate constitutive models for the respective materials involved in the interaction. Uwabe and Moriya (1988) confirmed the development of strain localization in their shaking table experiments. Therefore, for the sandy backfill, the localization based constitutive law (Hazarika and Matsuzawa, 1996) was adopted. For the tire chips cushion material, the model proposed by Youwai and Bergado (2003) was adopted. The selection of these two particular models was purely due to proximity of the respective materials' constitutive behavior to those represented by the two models referred here.

6.3 Interface modeling

The interface constitutive law was assumed to be bi-linear elasto-plastic obeying the Mohr-Coulomb failure criterion with zero cohesion. The reason for such assumption was mere simplicity in the modeling. And, in many geotechnical problems such assumption does not seem to yield a significant difference in the calculated results. The parameters of the interfaces were determined from the data based on direct shear tests performed on various interface conditions involving cushion materials, soils and structural materials (Hazarika et al., 2005; Karmokar et al., 2005). The interface between the caisson base and the foundation rubble was simulated by making use of the Eq. (9). For thin layer interface, Zaman et al (1984) suggested the thickness of the interface element to be 0.05 times the dimension of the adjacent soil element so that the numerical stability can be maintained with less margin of error. In the model described herein, since two different interfaces are involved, it is difficult to adopt any such general value for the thickness. Parametric studies are required for proper estimation of the respective thickness. However, for the sake of pure simplicity and also due to lack of such data, it was assumed here that the recommendation of Zaman et al (1984) is valid for such types of interfaces as well. Table 3 lists the basic material parameter values that were used (assumed) in the simulation. The following values of the *participation factors* were assumed for the caisson-cushion and cushion-backfill interfaces:

$$\alpha_1 = \beta_1 = 0.75; \alpha_2 = \beta_2 = 0.25; \alpha_3 = \beta_3 = 0.00 \cdot$$

Table 3 Material parameters used in the analyses

Parameters	Backfill & Foundation Soil	Tire chips Cushion
Young's modulus, E	26 MPa	2.6 MPa
Poisson's ratio, ν	0.30	0.20
Unit weight, γ	15.0 kN/m^3	7.0 kN/m^3
Angle of internal friction, ϕ	40 °	30 °

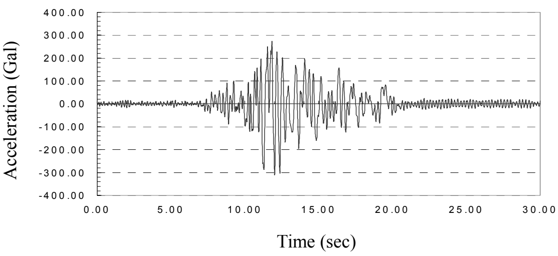
6.4 Analyses procedures

Analyses were conducted for two different backfill conditions (Refer to Fig. 5). One is the case in which the caisson was without any protective cushion, and the other is the case in which a cushion layer was sandwiched between the backfill soil and the caisson in order to enhance its seismic performance.

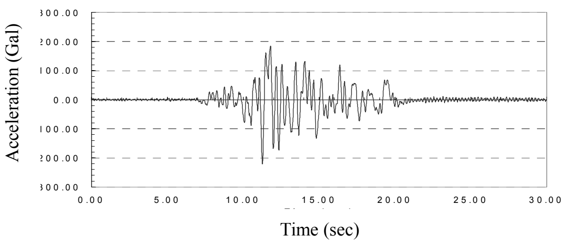
Static analyses were first performed under gravity loading to calculate the static at-rest earth pressure. Dynamic analyses were then performed, by imparting an actual earthquake motion at the fixed boundary of the simulated model (Fig. 32). The acceleration time history the 1968 Tokachi-Oki earthquake (Fig. 10b) was adopted in the analysis. The reason for adopting this motion was merely due to the fact that this is the de facto standard used frequently in the design of Port and Harbor facilities.

6.5 Analyses results

Fig. 33 shows the response accelerations of the caisson with sandwiched cushion at the elements near the top and bottom of the caisson-cushion interface (Interface I in Fig. 31). It can be observed that responses at the top and the bottom of the caisson are quite different. While the top experiences a high acceleration magnitude, the bottom experiences relatively low acceleration magnitude. Similar behavior was also observed in the case with no cushion, which was discussed in sub-section 3.1.



(a) Near the top of the caisson

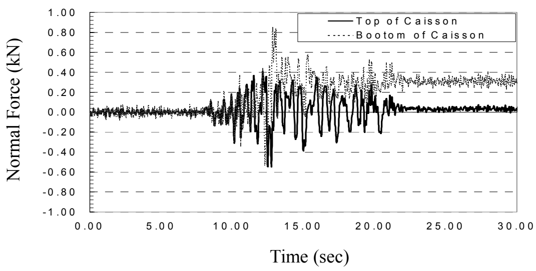


(b) Near the bottom of the caisson

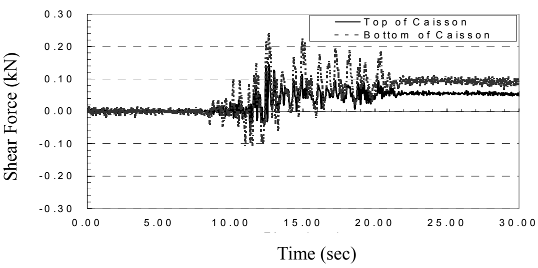
Figure 33 Response accelerations

The response of the interactive system at the caisson-cushion interface was also examined to see how the developed model could elucidate the soil-structure interaction phenomenon. Such responses are shown in Fig. 34. It can be seen that, both the normal and the shear force exhibit higher values at the top part. The normal force at the top, however, does not increase much, and drops to zero as compared to at the bottom. This can be attributed to the debonding (separation) tendency at the caisson-cushion interface. Such relative deformation at the interfaces, during the dynamic loading, can be explained well by the interface model described here.

Figure 35 shows a comparison of the resultant horizontal seismic earth pressure acting on the caisson. The values were obtained by summing up the nodal stresses of the elements at the caisson-cushion interface. It can be observed that the use of cushion could significantly reduce the seismic earth pressure acting on the wall. While the caisson without any protective cushion experiences high fluctuation of the earth pressure with a predominant peak, the earth pressure on the protected caisson stabilizes soon after reaching the peak. This tendency was also observed in the physical model test results, which was discussed in section 3.



(a) Normal force



(b) Shear force

Figure 34 Normal and shear forces at the caisson-cushion interface

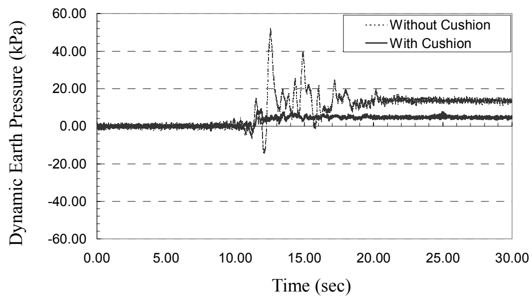


Figure 35 Resultant dynamic earth pressures

The dynamic earth pressures at the maximum inertia force are plotted against the wall height (Figure 36) to observe how the pressure distributes along the height for the two cases considered in the analyses. Only the values at some particular elements at the caisson-cushion interface were plotted. It can be seen that the distribution pattern is different for the two cases. Distribution for backfill without any cushion shows a nonlinear increase of the earth pressure with the wall depth. However, distribution for the backfill with cushion shows a maximum increase in the middle of the caisson height, and then a gradual decrease. This tendency demonstrates that the compressibility effect becomes dominant with the increase of depth, however, the dominancy wanes after reaching a certain depth. That depth may depend upon the height and rigidity of the structure as well as the rigidity and the thickness of the cushion material itself.

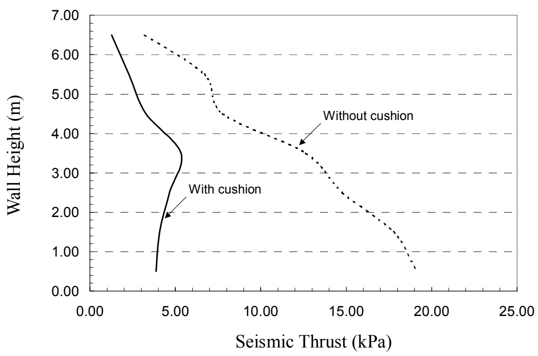


Figure 36 Seismic thrust distribution on the caisson

The numerically calculated distribution pattern displayed in Fig. 36, however, differs from the one obtained from the model test results (Fig. 17 in section 3). One reason for this may be due to the assumptions involved regarding the interface thickness and the participation factors in the model described in section 5. Parametric studies are needed for refinement of the numerical model to obtain reliable results that may be able to explain the experimental trend. Future research will be streamlined in that direction.

7. Conclusions

Use of sandwiched cushion (made out of recycled product such as tire chips) behind rigid and massive structures such as caisson quay wall results in the reduction of the seismic load and the residual deformation of the wall during the earthquake. Reduction of the load against structure implies lowering of the design seismic load, which in turn yields a slim structure with reduced material cost. Readily available tire chips, which are also relatively cheap, thus not only lead to an economic design, but also contribute to the satisfactory performance of structures by rendering flexibility and stability to structures during the earthquake. The benefit of the technique described here is not restricted to the newly constructed structures alone. The technique can also be applied for upgrading of (reinforcing) the existing structures, which do not satisfy the current seismic design criteria and, thus, run the risk of damages during the predicted devastating earthquakes such as Tokai earthquake and Tonankai-Nankai earthquake.

This research is a first step towards establishing the effectiveness of a newly developed seismic performance enhancement technique. Therefore, only a constant thickness cushion layer was considered here and hence the quantitative results described here are valid only for the particular thickness. Examination of the performance enhancement effects under different earthquake motions with constant value of cushion thickness has also revealed that the design cushion thickness may vary depending upon the level of the design ground motion. Performance-based design, which has been becoming the norm in many countries, will determine the final thickness. However, it is important to recognize that the stronger Level 2 ground motion will not necessarily solely dictate the final design. Further researches are necessary to clarify the influences of various other factors on the load reduction capacity, and to arrive at a cost-effective solution. These factors include the cushion thickness, density of the tire chips, the size of the chips particles, the rigidity of the cushion and the structure, etc. These will also determine the reduction capability of the earthquake-induced permanent displacements of the structures.

The seismic performance enhancement technique developed here is one way of contributing towards the material recycling of scrap tires. Such type of recycling not only relieves the burden on our environment (by reducing the level of CO_2 emission), but also can reduce the execution and construction cost of a project. Therefore, cost-performance benefit that can be achieved by the developed technique is potentially high. With growing emphasis on the industrial by-product in construction, and the problems associated with the use of scrap tires, this kind of novel construction technique, thereby, will not only lead to a cost-effective seismic design of structures, but also towards improvement of the seismic performances of existing structures. The application of the technique, thus, is expected to be a big boon towards a sustainable development of our environment.

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